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Large-scale in-situ test for mechanical characterization of soil–rock mixture used in an embankment dam

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A B S T R A C T

To improve the mechanical behavior of core wall in higher embankment dams, soil–rock mixture obtained by mixing rock blocks to the cohesive soil is used for the first time as the core wall material of the Nuozhadu embankment dam, China. The differences in the mechanical behavior of soil and soil–rock mixture samples are studied in depth. Two large-scale compaction test fields and a series of in-situ direct shear tests have been conducted on soil and soil–rock mixture samples. The mixing of rock blocks changes the deformation behavior of the sample. The existence of rock blocks makes the deformation modulus and the internal friction angle of soil–rock mixture greater than that of the soil sample, while decreasing its cohesive force.

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1. Introduction

Soil–rock mixture (S–RM) as a type of special geological body is abundant in nature, and is frequently used as a filling material in geotechnical engineering. For example, in embankment dams, which are more common worldwide, the core wall is one of the important parts that not only satisfies the requirement of seepage prevention but also has good mechanical strength. With the development of embankment dams around the world, their height continues to increase. To keep the deformation conformity between the rockfill of the dam and the core wall, and to decrease the arch effect in the core wall so that the stress state in the core wall can be improved and the probability of cracks occurring in the core wall reduced, the core wall material should have a higher deformation modulus.

Different from general soil (for example, clay, silt, and sand), S–RM contains a certain amount of oversize particles. The earliest work on S–RM in the laboratory was by Hall1, who focused on the development of a triaxial apparatus for testing large soil samples of at least 30.5 cm. Donaghe and Torrey2 studied the shear strength of S–RM using triaxial tests on 38.1 cm specimens, and found that the effective friction angle increased with increasing gravel contents. Other researchers3–9 concluded that the shear strength of clay–rock mixtures gradually increases with increasing percentages of floating particles in unsaturated clays. Using laboratory triaxial tests, Lindquist and co-workers10–12 studied the relationship between rock block proportion and shear strength of mélange. Through triaxial tests, Dupla et al.13 also indicated that the volumetric fraction of gravels is the main parameter of the elastic and the material failure characteristics of coarse-grained soils. From these studies, it was also concluded that the existence of oversize “rock blocks” in S–RM will influence the mechanical behavior of the material. Furthermore, most of the previous studies have focused on laboratory tests. As we know, it is very difficult to obtain undisturbed S–RM samples, and all laboratory tests are based on reconstructed ones. A further limitation of the laboratory conditions is that the size of the sample is usually less than 30 cm, and usually with a diameter of 15 cm. Based on the recommendation of ASTM, the maximum particle size for testing dimension is less than one-sixth of the sample's diameter. For example, the large testing equipment commonly used in the geotechnical engineering is less than 30 cm in diameter and the maximum particle size accommodated is less than 6 cm.

Rock blocks of S–RM in nature or in those used in the fills of high dams and other embankments are usually larger than the requirement for laboratory tests12. Therefore, scalping/replacing or scalping is commonly used in the processing of oversized particles. However, according to10, none of these methods yield satisfactory estimates of total stress strength parameters for the parent full-scale gradations of S–RM.

With the development of many types of large-scale engineering projects, there is an increasing need for progress in the knowledge of the mechanical behavior of S–RM. To compensate for the limitations of the laboratory studies of S–RM, more works on naturally formed S–RM is based on in-situ tests12. However,
because of the cost and difficulty of testing, S–RM used in the fills of high dams and other embankments has seldom been tested for shear resistance in-situ. Furthermore, mixing the rock blocks in clay as the core wall material is being used for the first time in China, and is seldom applied elsewhere in the world.

The gradation and content of oversized rock blocks in S–RM will influence the distribution of the meso-stress field, deformation and failure, and the macro-mechanical characteristics. To study the mechanical behavior of the matrix before and after mixing the rock blocks, and taking the core wall material of the Nuozhadu embankment dam as an example, we conducted large-scale in-situ direct shear tests. Most of the in-situ and laboratory tests can only provide the macro-mechanical properties and deformation behavior of geomaterials, and cannot easily reveal or difficult their meso-mechanic, deformation and failure mechanism. Therefore, in the companion paper, a set of numerical tests based on the discrete element method (DEM) are presented to study the meso-mechanical behavior of the soil before and after mixing the rock blocks.

2. Background of study

2.1. Description of the Nuozhadu dam

The Nuozhadu hydropower station is located within the middle and lower reaches of the Lancang River, in Yunnan province, China. Nuozhadu dam is a core rockfill dam (Fig. 1) with a height of 261.5 m. The maximum top width of the dam is 18 m, the slope ratio of the upstream and downstream is 1:1.9 and 1:1.0 respectively, and the maximum top width of the core wall is 10 m. Nuozhadu dam is the highest embankment dam in China, and the fourth highest in the world. Because of the strength and the deformation modulus of the rockfill composition, the main body of the dam is higher and the core wall material should also have a better deformation modulus so that it can coordinate the deformation of the adjacent rockfill. The generic type of core wall fill material is residual soil. Based on the laboratory tests, to satisfy the safety requirement of the dam, the original residual soil should be blended with the rock blocks up to a content of 35%, and the diameter of the particles should be less than 120 mm.

The lithology of the rock blocks used in S–RM is granite, with a uniaxial compressive strength of 69.4–165.2 MPa, and it is difficult to crush. Fig. 2 shows the typical particle-size distribution of the original residual soil and the residual soil mixing the rock blocks.

2.2. Preparation of the test field

To study the difference of the mechanical strength of the residual soil and the S–RM with mixing rock blocks, we carried out large-scale compaction tests for both materials. Each testing field is 30 m long and 20 m wide. The compaction pattern of the two testing fields is the same, all using a vibratory padfoot roller (working mass of 18,700 kg, exciting force of 380 kN, and height of padfoot of 10 cm). The thickness of the filling layer is approximately 35 cm, and the total rolling time of each layer is 12. After the 6th time has finished, the compacted density of the compacting layer is tested using a test pit (diameter of the pit is 30 cm). If the compacted density satisfies the designer requirement, we proceed with the remaining rolls. The total filling height of the compaction test is about 5 m. Fig. 3 shows the compaction test of the S–RM.

Fig. 4 shows the excavation section of S–RM after compaction, where the oversized rock blocks “float” uniformly in the compacted matrix clay with very few direct contacts between the rock blocks. The sample is denser, and the compaction test attained the ideal effect.

3. Large-scale in-situ test procedures

Despite there being many limitations to direct shear testing, including the non-uniformity of the stresses and strains within the box, owing to its simplicity and suitability for testing, the direct shear box is one of the most widely used devices for obtaining shear strength parameters (cohesive force and internal friction angle) for geotechnical material. After the preparation of the compaction field of soil and S–RM, four large-scale in-situ direct shear tests were carried out to understand the macro-mechanical behavior of the materials. The scale of the direct shear
test was 60 × 60 × 30 cm³ (Fig. 5). The test procedures used in this paper are described below.

3.1. Preparing the back pressure system

In general, the methods used for normal pressure of direct shear tests include, for example, lateral wall friction, ground anchor, and heaped load. We can select a suitable method based on the field conditions and engineering requirements. In this work, the heaped load method was used as the back-pressure system. I-beams are placed on the supporting columns, which lies on the two sides of the test sample, then sand bags are placed on the I-beams (Fig. 5a). The weight of the heaped load is approximately two times the normal load of the test.

3.2. Preparing the test sample

We placed the steel shear box on the ground at the testing site and pressed it into the testing soil vertically through a jack until the whole shear box was embedded in the soil. During the pressing process, the soil around the shear box was taken out continuously. In the end, a groove with about 50 cm was excavated around the test sample.

3.3. Preparing the reaction bearing of the shearing jack

To provide strong back force on the horizontal shearing jack, the reaction bearing of the jack was precast with concrete and a vertical jack connection with the back pressure system through a force transfer column was used to add vertical pressure on the concrete (Fig. 5b).

3.4. Installing the test equipment

After the above preparation, the bearing plate, sliding steel plate, jack and force transfer column were placed on the sample consecutively. The central axis of the jack should be consistent with that of the sample, and the jack must be perpendicular to the top surface of the sample so the normal stress of the jack can apply on the sample perpendicularly. To minimize the side friction.
effects of the shear box, the bearing plate and the upper shear box were connected together, so that they could move consistently in the vertical and horizontal directions.

Then the steel plate and jack were placed on the side of the shear box where the horizontal shear stress was applied, and to keep the horizontal shear stress applied on the sample horizontally, the medial axis of the jack had to lie in the medial plane of the sample.

When the normal and horizontal jacks were successfully placed, the measuring equipment was installed accordingly: an oil pressure gauge integrated with the jack was used for the pressure measurement of the jack and the dial indicators were used for the measurement of the horizontal displacements. All pressure and displacement measurements were recorded manually at every load step. Fig. 5(b) shows the final test equipment.

3.5. Consolidation

The normal stress for the test was divided into five loading steps, and each loading step was applied on the sample with mean time intervals of 5 min. When the normal stress reached the predetermined value and the vertical deformation rate was less than 0.01 mm/h, the consolidation was finished and the shear test could be carried out.

3.6. Shear

When the consolidation step was completed, the horizontal shear stress was applied step by step, and controlled so that the horizontal deformation rate was approximately 2 mm/(15–20)s. At each horizontal shear stress step, we checked the reading of the normal stress transducer and kept it at approximately the predetermined value. If the reading of the normal stress transducer deviated from the predetermined value, the vertical jack was adjusted to make its pressure return to the predetermined value.

After each horizontal shear stress step was finished and became stable, we recorded the reading of the horizontal pressure transducer and the dial indicators, and then applied the next shear step. Once the horizontal shear stress remained invariant over three to four consecutive loading steps, the test could be considered complete.

3.7. Test over and test results analysis

Once the test was complete, we dismantled the test equipment, recorded the final state of the shear zone, deformation characteristics and took photos, to be used for studying the deformation and failure mechanism of the sample. The horizontal shear stress versus horizontal displacement curves were produced and analyzed.

4. Analysis of test results

4.1. Characteristics of the test curves

Based on the field test results, Fig. 6 shows the horizontal shear stress and displacement curves of the direct shear tests of soil and S–RM with mixing rock block content of 35% under different normal stress conditions. According to the shear stress–shear strain curves of the soil (Fig. 6a), at the start of the test, the shear stress exhibits an approximate linear increase with the shear strain, reaching the peak strength and then becoming steady. As with the soil sample, in S–RM with mixing rock blocks, the shear stress is approximately linear with the shear strain at the beginning of the test, but before reaching the peak strength there is strain hardening, which is more obvious under higher normal stress (Fig. 6b). The rock block content of S–RM is 35%, and the rock blocks “float” in the compacted matrix clay. During the test, first, the “soil” of the S–RM tends to fail; then, with the increase of the shear strain the “floating” rock blocks will move, rotate, and some blocks may attach to each other, which will increase the shear strength of the sample. As a result, a strain hardening stage may occur before reaching peak strength. In this process, the internal fabric of the shear band will adjust step by step and become stable, and the shear stress will also reach a stable value (peak strength).

From the relationship between the ratio of shear stress/normal stress ($\tau/\sigma$) and the shear strain (Fig. 7), it can be seen that for both the soil and the S–RM sample the value of $R_{\tau/\sigma}$ will decrease with the increase in the normal stress. Under higher normal stress, the $R_{\tau/\sigma}$ value and the shear strength of S–RM is greater than that of the soil. Furthermore, the mixing of the rock blocks improves the deformation modulus of the sample.

4.2. Characteristics of shear strength

Based on the in-situ tests, Fig. 8 shows the relationship between the shear strength and the normal stress of the test samples. The shear strength parameters of the soil were $\phi = 13.6^\circ$ and $c = 173$ kPa. After mixing rock blocks, the shear strength
parameters of S–RM were $\phi = 20.8^\circ$ and $c = 138$ kPa. After mixing the rock blocks, the friction angle of the sample increases ($\Delta \phi = 7.2^\circ$), while the cohesiveness decreases ($\Delta c = -35$ kPa). This indicates that in the shearing process the movement, rotation and attachment of the rock blocks to each other will make the shear band uneven, which increases the friction angle of the sample. Furthermore, for the S–RM samples, the cohesive soil content decreases, which will decrease the cohesion of the entire sample.

The changes of the friction angle of the soil and S–RM sample (Fig. 9a) are similar with the tests of1. However, there is great a large difference in the variation of the cohesion (Fig. 9b), which may be caused by the differences in the “fine soil”. In this work, the soil is cohesive with lower permeability, higher strength and compactness, while the cohesion of the soil sample in1 is small. Furthermore, in this work the compactness of the S–RM is high, although the cohesion of the S–RM decreases by only approximately 20% of that of the soil.

4.3. Three-dimensional characteristics of the sliding surface

To analyze the shear zone characteristics of the sample, once the test was complete, the upper part of the sample was moved.
We refer to the cohesive force of S–RM sample before reaching peak strength. Rock blocks may in causes the sliding surface of S–RM sample is greater than that of the soil sample. However, the cohesive force of S–RM will be greater than that of the soil sample, while in the lower stress state the effect is opposite.

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Fig. 11. Sketch of the development of soil–rock mixture (S–RM) localization band. A-B is the preset shear plane, the arrow shows the moving direction of the rock blocks, and the dashed line is the actual localization band.

(a) soil sample: the sliding surface is basically flat, with only some local fluctuation, and the scratches that formed in the shearing process are obvious;

(b) S–RM sample: the fluctuation of the sliding surface is obvious, and the difference in the fluctuation may reach 3–4 cm (Fig. 10). The sliding surface goes round one side of the “rock” block (Fig. 11), and no crushing rock block was found in the test. Scratches are found in the “soil” and also in the rock blocks. This indicates that the rock blocks may be attached to the adjacent blocks, which may improve the strength of the sample.

5. Conclusions

Mixing rock blocks in clay as the core wall material of a high embankment dam is a new method for improving its mechanical behavior and stress state. However, prior to this work few studies have been published on the use of cohesive soil mixing rock blocks (S–RM) as the core wall material. In China, the Nuozhadu embankment dam is the first to use this type of material in the core wall. Two large-scale compaction test fields and a series of in-situ direct shear tests were carried out to investigate the mechanical behavior of cohesive soil sample before and after the mixing of the rock blocks. The mixing of rock blocks changes the deformation behavior of sample, the deformation modulus of S–RM is higher than that of the soil sample; and there is a strain-hardening stage for the S–RM sample before reaching peak strength. Rock blocks may influence the formation of the localization band, which causes the sliding surface of S–RM to fluctuate more than that of the soil sample. For the higher strength of the mixed rock blocks, although there is obvious attachment between adjacent rock blocks, no crushing rock block was found in S–RM. The internal friction angle of the S–RM sample is greater than that of the soil sample. However, the cohesive force of S–RM is less than that of the soil. Therefore, in the higher stress state the shear strength of S–RM will be greater than that of the soil sample, while in the lower stress state the effect is opposite.

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